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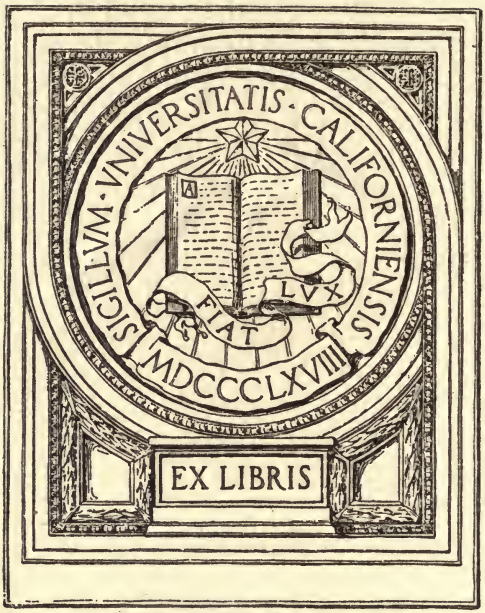
CALCULATION OF COLUMNS

THEODOR NIELSEN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CALCULATION OF COLUMNS

CALCULATION OF COLUMNS

A PRACTICAL APPLICATION OF THE THEORY

BY

THEODOR NIELSEN

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PREFACE

THE object of this treatise is, to a great extent, to show the results of the researches of Professor A. OSTENFELD, of Copenhagen, the application of which affords rapid means of arriving at a reliable cross section of a mild steel column, and in general the strength of any column. It is supplemented with tables and with practical information dealing with the subject, which has been gathered by the author from time to time, and it is an extension of a paper (in the metric system only), read in Spanish before the Engineering Section of the Scientific Congress held in Buenos Aires in July 1910, to which the Author was the official representative of the Danish Government.

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NOTATIONS USED.

P = Force.

A = Sectional Area.

I = Moment of Inertia.

$r = \sqrt{\frac{I}{A}}$ = Radius of Gyration.

L = Actual Length.

l = "Free" Length (according to the method of fixing).

$x = \frac{l}{r}$ = Ratio of Free Length to Radius of Gyration.

e = Distance from Neutral Axis to extreme Fibre.

$R = \frac{I}{e}$ = Modulus of Section or Geometrical Moment of Resistance.

$k = \frac{r^2}{e} = \frac{R}{A}$ = Radius of Core (kernel).

E = Modulus of Elasticity.

S = Breaking Stress per unit area.

s = Stress per unit area.

B = Compressive Stress per unit area of a short bar.

C and c are constants depending upon the material.

CALCULATION OF COLUMNS

Free Length for Centrally Loaded Columns.

EXPERIMENTS and calculation show that the centre line of a column will, if deflected, assume a wave line, and the free length will be considered as the distance between two consecutive points of contraflexure, this being the length of a column with ideal pin ends. The first step in all column calculations is to deduce the free length from the actual length, and it is also the weak point of such calculations, as the free length depends upon the nature of the fixing of the ends (which may even cause some eccentricity of the loading). In case where any doubt exists it is better to assume a greater free length, say full length, of the column even if the ends are held.

As a guide the following will prove useful, but much depends upon individual judgment.

Both ends free : the free length $l = L$ is purely ideal, but very often allowed.

Pin ends : $l = 0{\cdot}8$ L deduced from U.S. tests by Prof. Ostenfeld.

Flat ends : $l = 0{\cdot}65$ L to $0{\cdot}75$ L in general, but for Phoenix, Z-bar and similar columns $l = 0{\cdot}6$ L deduced from U.S. tests by Prof. Ostenfeld. For wooden posts, $l = 0{\cdot}75$ L recommended by Prof. Tetmajer.

Both ends firmly held : $l = 0{\cdot}5$ L is a theoretical value ; Prof. Claxton Fidler recommends for practice $l = 0{\cdot}6$ L.

Foot firmly held, top guided : $l = 0{\cdot}7$ L (theoretically $\frac{L}{\sqrt{2}}$).

Foot firmly held, top free : $l = 2$ L.

Lattice Girders.—Each compressive member will have a tendency (1) to deflect in the plane of the girder, and (2) normal to this plane. Generally the free length will be different in the two cases, and nearly always the moment of inertia is different, a greater moment corresponding to the greater l normal to the plane. It is not always possible to guess what is weakest, and calculation should then be made for the two planes normal to each other corresponding to the two principal axis of the section.

A compressive boom may in the plane of the girder have a tendency to form a wave line with points of reverse curvature in the apices, hence, say l = distance of apices. In the plane normal to the girder the same will be the case if there is a good cross and diagonal bracing, otherwise there will be a tendency to form a longer wave, say l = double the distance of the apices, an assumption allowed by Prof. Lütken of Copenhagen.

A strut of length L will get very little help from the booms against deflection normal to the plane of the girder, hence say l = L, only when there is a good cross bracing a shorter length might be allowed. In the plane of the girder the heavy booms will give a great support to the ends of the light strut. Prof. Lütken allows for riveted ends $l = 0{\cdot}6$ L, except for the end strut where l = L, as this strut behaves as a boom. If, as in a double Warren lattice, a tie of the same numerical stress is riveted to the centre of the strut he allows in the plane of the girder $l = 0{\cdot}4$ L.

In a multiple lattice Prof. Tetmajer allows $l = 1{\cdot}8$ times length of mesh. This figure is to the safe side of the results of calculations by Prof. Schüle in Zürich, concerning tests to destruction of an old 28 metres railway bridge that broke in the lattice bars of channel irons riveted to ties of flats. For flat lattice bars Messrs. Laissle and Schübeler, of Würtemberg (Bau der Brückenträger, 1871), assume that the ends may be considered theoretically held, and that, as the tension bar of the same section tends to stiffen the compression bar, this bar may be calculated as if it had double its actual moment of inertia. Based on these assumptions, the author

has calculated the free length of a flat lattice bar to be 0·35, say $\frac{1}{3}$ of its length between nearest supports by booms or stiffeners.

Factors of Safety and Unit Stresses.

Prof. Ostenfeld recommends the following:

For **mild steel** and **wrought iron** a factor of safety from 3 to 5.

For buildings where the stresses have been very carefully taken out, considering the uneven distribution of the load, he allows a factor of safety of 3 for mild steel and a unit stress of 0·95 t/cm² = 6·0 tons/□″ = 13 500 lbs./□″ on short bars. (1 t = 1000 kg = 0·984 English tons = 2205 lbs., 1 t/cm² = 6·35 tons/□″ = 14 223 lbs./□″.)

For buildings where the stresses have been taken out in the ordinary way, allowing a super-load per unit area, he allows a factor of safety of 4, or a unit stress of 0·7 t/cm² = 4·4 tons/□″ = 10 000 lbs./□″ on short bars.

For **cast iron** he uses a factor of safety from 8 to 10, usually the latter.

For **wood** he allows a factor of safety from 5 to 8. For European pine, when using Tetmajer's straight line continued by Euler's formula, he uses a factor of safety of 6.

Columns with Free Ends and Centrally Loaded.

It is now universally recognised that **Euler's** formula gives good results in practice for long columns. This formula is

$$\mathbf{P} = \frac{\pi^2 \mathbf{E I}}{l^2} \quad . \quad . \quad . \quad (1)$$

A more modern and often more convenient form of it is as follows:

Introducing $\mathbf{P = A S}$ . . . (2)

and $I = A r^2$ gives

$$\mathbf{S} = \frac{\pi^2 \mathbf{E}}{\left(\frac{l}{r}\right)^2} \text{ or } \frac{\pi^2 \mathbf{E}}{\mathbf{x}^2} \quad . \quad . \quad (3)$$

$\pi^2 = 9{\cdot}870$, often rounded off to 10.

Formulæ (2) and (3) are more convenient to use than (1), provided that tables are worked out for (3) and for r. The latter of course can be obtained from any English and American Standard Section Table, but r is not found in the German Standard Table, therefore it is given here, pages 34 and 35.

It must be remembered that for any column calculation I and r must be taken relative to the direction in which there is a curvature tendency (i.e. the neutral axis must be taken normal to this plane), hence in most cases the minimum values will be necessary in calculations: (note that for compound sections usually the min. r for the section equals the max. r for the single bar).

Plotting $x = \frac{l}{r}$ as abscissa and S as ordinate (see Figs. 1 to 3) we obtain the Euler Curve.

For **short columns** Euler's formula is dangerous as the results are far too high.

The late **Prof. J. B. Johnson**, of the U.S. (see Johnson, Bryan and Turneaure: Modern framed structures), proposes for short columns to use a Parabola which has the axis of ordinates for axis, and is tangent to the Euler curve, all plotted as mentioned above.

J. B. Johnson's formula is:

$$\mathbf{S = B - C\,x^2} \quad . \quad . \quad . \quad (4)$$

At the tangent point (3) and (4) must give the same value of S hence

$$\frac{\pi^2 E}{x^2} = B - C x^2$$

or

$$B x^2 - C x^4 = \pi^2 E \quad . \quad . \quad (5)$$

equally the same value of $\frac{d S}{d x}$,

hence

$$-\frac{2 \pi^2 E}{x^3} = -2 C x$$

or

$$C x^4 = \pi^2 E \quad . \quad . \quad (6)$$

The addition of (5) and (6) gives

$$B x^2 = 2 \pi^2 E$$

hence we obtain the junction of Johnson and Euler's formulæ at

$$\frac{l}{r} \text{ or } x = \sqrt{\frac{2 \pi^2 E}{B}} \quad . \quad . \quad (7)$$

$$2 \pi^2 = 19{\cdot}74$$

substituting x in (6) gives

$$C \frac{4 \pi^4 E^2}{B^2} = \pi^2 E$$

hence

$$C = \frac{B^2}{4 \pi^2 E} \quad . \quad . \quad . \quad (8)$$

$$4 \pi^2 = 39{\cdot}48.$$

As J. B. Johnson points out, B must be practically the yield point or, for material without it, the compressive stress.

Having B it is easy to find C by (8) and the limit x by (7); thus we can obtain reasonable column formulæ by (4) and by (3), as continuation, with very little work.

We can find B, even without a testing-machine, from the load that produces yielding (or first crack), and by noting deflections caused by certain loads on a small square beam, we can calculate E (to the safe side). This is particularly useful for timber, and an example shown later on for pitch pine will illustrate the formation of such formulæ.

Mild Steel and Wrought Iron. Central Load.

Exhaustive tests were made by the late Prof. L. v. Tetmajer of Vienna, when he was in Zürich, for columns with free ends (using steel points). Based on these tests, Prof. Ostenfeld calculated B and C in (4) by means of the method of the minor squares to be in kg/cm² (1 kg/cm² = 0·00635 tons/□″ = 14·22 lbs./□″) for

Mild steel . .	B = 2724	C = 0·086.
Wrought iron .	B = 2581	C = 0·087.

It may be added that for Prof. Tetmajer's tests the mean values of the yield points were for mild steel 2800, and for wrought iron 2450 kg/cm², hence very near the values of B given above, agreeing with J. B. Johnson's assumption.

These values did not exactly fit the Euler curves for **mild steel** with **E** = 2 150 000 kg/cm² = **2150 t/cm²** = **13 600 tons/□″** = 30 600 000 lbs./□″, and **wrought iron** with **E** = 2 000 000 kg/cm² = **2000 t/cm²** = **12 700 tons/□″** = 28 400 000 lbs./□″ as used by Prof. Tetmajer, therefore Prof. Ostenfeld assumed B as above, and calculated C by (8) obtaining for mild steel 0·0874, and for wrought iron 0·0844, or very little difference, which shows that Prof. J. B. Johnson's formula is most reasonable. Thus, to suit Euler with above values of E, the Johnson formula (4) $S = B - Cx^2$ gives the **Johnson-Ostenfeld** formula for

Mild Steel . . . $\mathbf{S = 2724 - 0{\cdot}0874 \left(\frac{l}{r}\right)}$

kg/cm². . (9) limit $\frac{l}{r} = 125$

Wrought Iron . . $\mathbf{S = 2581 - 0{\cdot}0844 \left(\frac{l}{r}\right)^2}$

kg/cm² . . . (10) limit $\frac{l}{r} = 124$.

These figures do not agree too well with Prof. Johnson's own figures, but are preferable, as they give the mean results of Prof. Tetmajer's excellent tests.

The author has worked out the accompanying table based on (9) and (10) as far as $\frac{l}{r} = 120$, from $\frac{l}{r} = 125$ to 200, the latter being beyond what is desirable in practice; the metric values have been copied from Prof. Tetmajer's book, "Angewandte Festigkeitslehre" of 1904.

Ex. 1.—A mild steel, B. S. H-beam 9″ × 7″ × 58 lb. of 20′ 0″ length with a substantial base carries a ceiling. Find safe load for a factor of safety 4.

L = 20′ 0″ = 240″; foot held and top guided; hence free length $l = 0{\cdot}7 \times 240 = 168$″.

then $r = 1''{\cdot}64$; $\frac{l}{r} = \frac{168}{1{\cdot}64} = 102$ gives by interpolation in Table I. (page 32) breaking load per unit area

$$S = 11{\cdot}5 \text{ tons}/\square''.$$

$$A = 17{\cdot}1\ \square''.$$

Breaking load $A\,S = 17{\cdot}1 \times 11{\cdot}5 = 197$ tons.

Safe load $\frac{197}{4} = 49$ tons.

Ex. 2.—Two German standard channel bars N.P. No. 20 are laced together and form a 3·24 m long strut in a bridge girder, and is stressed by 33·8 t. Find the factor of safety.

$$L = 324 \text{ cm.}$$

Free length $l = 0{\cdot}6 \times 324 = 194$ cm.

Min. r for the strut is max. r for the channel ; hence $r = 7{\cdot}71$ cm. (see page 34).

$$\frac{l}{r} = \frac{194}{7{\cdot}71} = 25.$$

By Table I. $S = 2.67$ t/cm².

$A = 2 \times 32{\cdot}2 = 64{\cdot}4$ cm².

Breaking load $A\,S = 64{\cdot}4 \times 2{\cdot}67 = 172$ t.

Factor of safety $\frac{172}{33{\cdot}8} = 5{\cdot}1$.

If the ends were not well stiffened or the booms not stiff, it would be wise to take

$$l = L = 324 \text{ cm.}$$

$$\frac{l}{r} = \frac{324}{7{\cdot}71} = 42.$$

From Table I. $S = 2{\cdot}57$ t/cm².

Breaking load $A\,S = 64{\cdot}4 \times 2{\cdot}57 = 165$ t.

Factor of safety $\frac{165}{33{\cdot}8} = 4{\cdot}9$.

It will be noted that from (9) $\frac{2724}{0{\cdot}0874} = 31\,200$, and from

(10) $\frac{2581}{0{\cdot}0844} = 30\,600$ are both very nearly equal to 30 000, and **Prof. Ostenfeld** uses an approximation

$$\mathbf{S = \left(1 + \frac{X^2}{30\,000}\right) B} \quad . \quad . \quad (11)$$

where $x = \frac{l}{r}$ and B is the breaking load of a short bar. Formulæ (9) and (10) agree very favourably with (11) when using for the latter :

Mild Steel. $\mathbf{B = 2{\cdot}8}$ **t/cm²** $= \mathbf{17{\cdot}8}$ **tons/□″** $= 40\,000$ lbs./□″

Wrought Iron. $\mathbf{B = 2{\cdot}6}$ **t/cm²** $= \mathbf{16{\cdot}5}$ **tons/□″** $= 37\,000$ lbs./□″

As working stress equals S or B divided by a factor of safety it is seen that

$$\mathbf{F = 1 + \frac{x^2}{30\,000}} \quad . \quad . \quad . \quad . \quad (12)$$

is simply a **Stress Factor** giving the working stress of a column with $x = \frac{l}{r}$ from that of a short bar.

As a stress factor is very convenient in practice the following shows how the author has obtained a corresponding factor to suit Euler's formula.

On comparing (11) and (4) it is seen that $C = \frac{B}{30\,000}$ and substituting this in (8) we have

$$\frac{B}{30\,000} = \frac{B^2}{4\pi^2 E}$$

hence

$$B = \frac{4\pi^2 E}{30\,000}.$$

For mild steel with $E = 2150$ t/cm² this gives $B = 2{\cdot}83$ t/cm² and for wrought iron with $E = 2000$ t/cm² we obtain $B = 2{\cdot}63$ t/cm² (these values are very near the $2{\cdot}8$ and $2{\cdot}6$ given above, or $2{\cdot}72$ and $2{\cdot}58$ from (9) and (10)). By intro-

ducing the values thus found in formula (7) we obtain for both mild steel and wrought iron a transit from Johnson to Euler at $\frac{l}{r} = 122\frac{1}{2}$ (or very near the 125 and 124 given by (9) and (10), the exact formulæ).

Now using the factor F corresponding to Euler the breaking load FB is equal to S in formula (3) hence

$$FB = \frac{\pi^2 E}{x^2} \qquad Fx^2 = \frac{\pi^2 E}{B}$$

Substituting the corresponding values recently found gives for both mild steel and wrought iron $\frac{\pi^2 E}{B} = 7500$. Hence the stress factor for Euler becomes

$$F = \frac{7500}{x^2} \quad . \quad . \quad . \quad . \quad . \quad (13)$$

Table I. gives the stress factors found from (12) and (13), and for working stress s of a short bar the column can be stressed to Fs.

It should be carefully noted, when using a certain working stress, that it gives a suitable factor of safety on B as given above. Thus using the ordinary 5 tons/□″ the factor of safety $= \frac{17{\cdot}8}{5} = 3{\cdot}6$, or less than is generally thought, as we are accustomed to think of ultimate crushing strength instead of stress at the yield point.

The following table shows a series of values obtained by using F and B given in t/cm² compared with the exact values from (9), (10) and (3) partially copied from Table I. It will be seen that the approximation is satisfactory.

$\frac{l}{r}$	Mild Steel		Wrought Iron	
	FB = F × 2·8	Formulæ (9) and (3)	FB = F × 2·6	Formulæ (10) and (3)
0	2·80	2·72	2·60	2·58
50	2·57	2·51	2·38	2·37
100	1·87	1·85	1·73	1·74
150	0·932	0·943	0·865	0·877
200	0·524	0·531	0·486	0·494
300	0·233	0·236	0·217	0·219
400	0·131	0·133	0·122	0·123

Ex. 3.—A 15′ long strut formed of two braced B.S. channels 5″ × 2½″ × 11 lb., and held by pins, is stressed to 4 tons/□″ for a short bar. Find the safe load.

$L = 180''$; $l = 0{\cdot}8 \times 180 = 144''$.

$r = 1{\cdot}94''$ (which equals max. r for a channel).

$$\frac{l}{r} = \frac{180}{1{\cdot}94} = 93.$$

From Table I., $F = 0{\cdot}711$.

$A = 2 \times 3{\cdot}23 = 6{\cdot}46$ □″.

Safe load $F s A = 0{\cdot}711 \times 4 \times 6{\cdot}46 = 18{\cdot}3$ tons.

Ex. 4.—A 6 metres long stanchion formed of two German standard joists N.P. No. 20, with stiffened bedplate, is stressed in short bar to 0·7 t/cm². Find the safe load.

$L = 600$ cm, $l = 0{\cdot}7 \times 600 = 420$ cm.

$r = 8{\cdot}00$ cm (equal to max. r for joist).

$$\frac{l}{r} = \frac{420}{8{\cdot}00} = 52.$$

From Table I. $F = 0{\cdot}910$.

$A = 2 \times 33{\cdot}4 = 66{\cdot}8$ cm².

Safe load $F s A = 0{\cdot}910 \times 0{\cdot}7 \times 66{\cdot}8 = 42{\cdot}5$ t.

To design an economical column usually requires a good deal of trial and error; it can, however, by using Prof. Ostenfeld's deductions, be made an extremely easy work for mild steel and wrought iron. This method gives the satisfaction that any trial approaches the result, and the second trial usually gives a satisfactory result.

He obtains his formula in the following way.

The working stress s of a column, corresponding to $\mathbf{s_0}$ **in a short bar** is by formula (11)

$$s = \left(1 - \frac{\left(\frac{l}{r}\right)^2}{30\,000}\right) s_0$$

He then introduces the **area $\mathbf{A_0}$ of a short bar** corresponding to a load P.

$$\mathbf{A_0 = \frac{P}{s_0}} \quad . \quad . \quad (14)$$

and Prof. Claxton Fidler's

$$\mathbf{Z = \frac{A}{r^2} = \frac{A^2}{I}} \quad . \quad . \quad (15)$$

The full area and the full moment of inertia must be used in (15); it would not be correct to work to one of the component members as used when determining the radius of gyration.

Substituting $r^2 = \frac{A}{z}$ from (15) and $s_0 = \frac{P}{A_0}$ from (14) into above expression for s gives

$$\frac{P}{A} = s = \left(1 - \frac{\left(\frac{z\,l^2}{A}\right)}{30\,000}\right)\frac{P}{A_0}$$

from which

$$A_0 = A - \frac{z\,l^2}{30\,000}$$

from this we obtain **Prof. Ostenfeld's formula**

$$\mathbf{A = A_0 + \frac{z\,l^2}{30\,000}} \quad . \quad . \quad (16)$$

If the length is l_1 metres and the areas are in square centimetres, the formula reduces itself to

$$\mathbf{A = A_0 + \tfrac{1}{3}\,z\,l_1^2} \quad . \quad . \quad (16)\text{A.}$$

Formula (16) gives the column area A_0 wanted for a short bar plus an increment $\frac{z\,l^2}{30\,000}$ to obtain stiffness, and he points out that z gives a measure for the economy of the section; the smaller z is, the less increment is wanted.

Formula (16) holds good up to $\frac{l}{r} = 122\frac{1}{2}$; at this limit we have

$$z\,l^2 = \frac{A\,l^2}{r^2} = A\left(\frac{l}{r}\right)^2 = A \times 122{\cdot}5^2 = 15\,000\,A.$$

Substituting this in (16) we have

$$A = A_0 + \frac{15\,000\,A}{30\,000} = A_0 + \tfrac{1}{2}A,$$

hence for the limit

$$\mathbf{A = 2\,A_0} \quad . \quad . \quad . \quad (17)$$

This limit is obtained when the stiffness increment is equal to what is required for a short bar.

Should the above ratio $\frac{l}{r}$ be exceeded, then Euler must be used, and the solution is quickly obtained. The formula (1) for the breaking load P gives

$$\mathbf{I} = \frac{\mathbf{P}\,\mathbf{l}^2}{\pi^2\,\mathbf{E}} \quad . \quad . \quad . \quad (18)$$

$\pi^2 = 9{\cdot}870$ or about 10.

It must be remembered that P equals the safe load wanted, multiplied by the factor of safety.

In using Ostenfeld's formula (16) or (16)A, the first trial is made by using a tabular value of z, which keeps fairly constant for similar sections (and exactly so for geometrical similarity). The value of z can be worked out for any section, but for a great many that are most frequently required in practice the values are given in the following table which is extended from one given by Prof. Ostenfeld.

Section	Z	Remarks
O	12·6	Round bar (exactly: $z = 4\pi$).
	0·63	$t = 0{\cdot}05\,r$, where r is mean radius of tube and t the thickness of the metal.
,,	1·25	$t = 0{\cdot}10\,r$.
,,	2·50	$t = 0{\cdot}20\,r$.
,,	3·66	$t = 0{\cdot}30\,r$.
□	12	Square bar.
b h	$\frac{12\,b}{h}$	Flat bar. b is greater than h.
$h \llcorner t$	8·4	Equal angle and $t = \frac{1}{6}\,h$.
,,	7·2	,, ,, ,, $t = \frac{1}{7}\,h$.
,,	6·3	,, ,, ,, $t = \frac{1}{8}\,h$.
,,	5·7	,, ,, ,, $t = \frac{1}{9}\,h$.
,,	5·0	,, ,, ,, $t = \frac{1}{10}\,h$.

Section	Z	Remarks
	7·1	4 equal angles riveted together, $t = \frac{1}{8} h$.
,,	5·8	,, ,, ,, ,, $t = \frac{1}{10} h$.
,,	5·0	,, ,, ,, ,, German standard.
,,	4·0	,, ,, ,, ,, ,, ,, 1 cm. clearance.
	3·3	2 equal angles with tie plates, $t = \frac{1}{8} h$.
	4·3	2 equal angles, $t = \frac{1}{8} h$.
	7·6	T bar with $b = 2\,h = 12\,t$.
,,	5·3	,, ,, $b = h = 8\,t$.
	8·6	2 T bars riveted together, $b = 2\,h = 12\,t$.
,,	4·2	,, with distance pieces of sufficient thickness.
	6	Channel bar, British standard, $h = 2\,b$.
,,	9	,, ,, but $h = 3\,b$.
,,	7	German standard channel bar.
		Braced channel bars of sufficient distance to bring max. r into play :
][	1·7	British standard with $h = 2\,b$.
,,	1·0	,, ,, ,, $h = 3\,b$.
,,	1·2	German standard in proper distance.
,,	6·0	,, having only 1 cm. clearance.
I	6	H beam, British standard, with h equal or less than 10″.
,,	10	,, ,, ,, ,, h greater than 10″.
,,	10	German standard.
,,	3	Differdange broad flange beam, h being 24 to 34 cm.
,,	4	,, ,, ,, ,, ,, 36 to 47½ cm.
I I	2	British standard H beams in sufficient distance, h equal or less than 12″.
,,	1	British standard H beams in sufficient distance, h greater than 12″.
,,	1 to 1·1	German standard H beams in sufficient distance.
	1·5	4 Phoenix bars, British standard.
,,	1·8	,, ,, German ,,

Ex. 5.—A strut of $l = 10' 0'' = 120''$ formed of one British standard H beam is required to carry 30 tons with a factor of safety 4. Find the section required.

As B = 17·8 tons/□″ the stress must be

$$s_0 = \frac{17 \cdot 8}{4} = 4 \cdot 45 \text{ tons/□″}$$

and by Prof. Ostenfeld's formula (16)

$$A_0 = \frac{P}{s_0} = \frac{30}{4 \cdot 45} = 6 \cdot 74 \text{ □″}.$$

Try a section with $z = 6$.

$$\frac{z l^2}{30\,000} = \frac{6 \times 120^2}{30\,000} = 2 \cdot 88.$$

$$A = A_0 + \frac{z l^2}{30\,000} = 6 \cdot 74 + 2 \cdot 88 = 9 \cdot 62 \text{ □″}.$$

An 8″ × 6″ × 35 lb. H beam has 10·29 □″ area.

Prof. Ostenfeld's check:

By (15) $z = \frac{A^2}{I} = \frac{10 \cdot 29^2}{17 \cdot 95} = 5 \cdot 90.$

(This really settles the question, as $z = 6$ was assumed; but, to clearly show the method, the full check will be taken.)

$$\frac{z l^2}{30\,000} = \frac{5 \cdot 90 \times 120^2}{30\,000} = 2 \cdot 83.$$

$$A = A_0 + \frac{z l^2}{30\,000} = 6 \cdot 74 + 2 \cdot 83 = 9 \cdot 57 \text{ □″}$$

is wanted or less than there is in the joist, hence the column is safe, and no other commercial section suits the area required, therefore no further trial is wanted.

As a further check the actual factor of safety will now be found

$$r = 1 \cdot 32''; \qquad \frac{l}{r} = \frac{120}{1 \cdot 32} = 91.$$

From Table I., S = 12·7 tons/□″.

$$\text{Factor of safety} = \frac{A\,S}{P} = \frac{10{\cdot}29 \times 12{\cdot}7}{30} = 4{\cdot}36.$$

Ex. 6.—A 7·4 m long tubular column with ball bearings is required to carry 65 t and to be formed of 4 German standard Phœnix bars riveted together, and to be stressed to 0·7 t/cm². Find the section required.

$$A_0 = \frac{P}{s_0} = \frac{65}{0{\cdot}7} = 92{\cdot}9 \text{ cm}^2.$$

By (16)*a* $\frac{1}{3}\,z\,l_1^2 = \frac{1}{3} \times 1{\cdot}8 \times 7{\cdot}4^2 = 32{\cdot}9$ cm².

$A = A_0 + \frac{1}{3}\,z\,l_1^2 = 92{\cdot}9 + 32{\cdot}9 = 125$ cm².

4 Phœnix bars No. 12½ × 10 mm. body have area = 129 cm².

$$\text{Check: } z = \frac{A^2}{I} = \frac{129^2}{12\,200} = 1{\cdot}37.$$

$\frac{1}{3}\,z\,l_1^2 = \frac{1}{3} \times 1{\cdot}37 \times 7{\cdot}4^2 = 25{\cdot}0$ cm²

$A = A_0 + \frac{1}{3}\,z\,l_1^2 = 92{\cdot}9 + 25{\cdot}0 = 118$ cm², hence safe.

However 4 bars of No. 10 × 12 mm. having 120 cm² might be tried.

$$z = \frac{A^2}{I} = \frac{120^2}{7480} = 1{\cdot}92.$$

$\frac{1}{3}\,z\,l_1^2 = \frac{1}{3} \times 1{\cdot}92 \times 7{\cdot}4^2 = 35{\cdot}0$.

$A = 92{\cdot}9 + 35{\cdot}0 = 128$ cm², hence they are not strong enough.

Ex. 7.—A mild steel column with 6 m free length is required to carry 4·0 t and to be formed of one German standard channel. Factor of safety 4. Find the section required.

$B = 2{\cdot}8$ t/cm², hence $s_0 = \frac{2{\cdot}8}{4} = 0{\cdot}7$ t/cm².

$$A_0 = \frac{P}{s_0} = \frac{4{\cdot}0}{0{\cdot}7} = 5{\cdot}7 \text{ cm}^2.$$

$\frac{1}{3}\,z\,l_1^2 = \frac{1}{3} \times 7 \times 6^2 = 84$ cm².

This increment is so much larger than A_0 that there is no doubt that Euler's formula must be used.

Breaking load P = 4 × 4·0 t = 16·0 t.

By formula (18) with $l = 600$ cm.

$$I = \frac{P l^2}{\pi^2 E} = \frac{16{\cdot}0 \times 600^2}{9{\cdot}87 \times 2150} = 271.$$

Channel N.P. No. 26 with I = 317 must be used as the next lower channel, No. 24 only has I = 248.

Note.—Examples 6 and 7 have been taken from Prof. Ostenfeld's "Teknisk Elasticitetslære" of 1905.

Built Columns. Central Load.

When a column is built up of sectional members held together with lacing, it is very tempting to consider each sectional member separately as a column of a length equal to the distance between the points held, similar to the method used for calculating the boom of a girder. This is, however, a dangerous practice, as tests prove that nothing like the distance of the held points that appeared safe from such a calculation would be permissible in practice.

Prof. Tetmajer, who made numerous tests with columns formed of 2 or 4 mild steel angle bars (cross section ⌐└ or ╣╠) held together with tie plates, found that no weakening was produced when the area of the rivet-holes did not exceed 12 per cent. of the cross sectional area, nor, when the tie plates were not spaced at a greater distance than 50 times the least radius of gyration of one of the angle bars.

Until further tests are made, Prof. Ostenfeld and L. Vianello, C.E., recommend that for built struts not more than 50 times the min. radius of gyration of the single bar should be allowed as the distance of side bracing for mild steel. Proportioning by the yield points gives the figure as 45 for wrought iron.

The late engineer, L. Vianello, in his book "Der Eisenbau" of 1905 works out in the following way a reasonable and plain method for calculating the bracing of a mild steel built column.

Generally, l is smaller than 105 r, hence Prof. Tetmajer's

formula $3 \cdot 1 - \frac{1}{88} \frac{l}{r}$ t/cm² (see formula (21), page 20) can be used for breaking load. If the distance of the centres of gravity of the sectional bars, or sets of bars, is h, we have approximately $r = \frac{1}{2} h$. Allowing the 3·1 t/cm² as the yield point, then the $\frac{1}{88} \frac{l}{r} = \frac{1}{44} \frac{l}{h}$ must be due to the bending effect alone. The elastic line of a column coincides fairly with that of an evenly loaded beam ; for the latter, the moment at the centre is $M = \frac{1}{8} p l^2$, and the corresponding stress in the sectional bars will consequently be $s = \frac{M}{h A} = \frac{p l^2}{8 h A}$, where A is the area of one of each of the two bars, or sets of bars. At the moment the column deflects we must have equality of the stresses found above, or

$$\frac{1}{44} \frac{l}{h} = \frac{p l^2}{8 h A} \qquad \text{hence } p = \frac{2 A}{11 l}$$

and the corresponding shear at the end must be $Q = \frac{1}{2} p l = \frac{A}{11}$ at the moment of yielding. This corresponds to breaking, but it is no use to design the bracing stronger than what corresponds to the safe load of the column.

Hence a smaller value of Q can be worked to, and allowing a working stress of 1 t/cm² against the 3·1 breaking load, the design of the bracing can be based on a value 3 times smaller than Q from above.

Therefore Vianello proportions the bracing to the crosswise end shear

$$Q = \frac{A}{33} \text{ metric tons} \quad . \quad . \quad (19)$$

where A is the area of a sectional bar, or set of bars, in cm².

For the area A_1 square inches we get

$$Q = 0 \cdot 193 \, A_1 \text{ English tons} \quad . \quad (19)A$$

Vianello says that, working to the safe side, Q can be considered constant up to $\frac{1}{6}$ of the length from each end of the column and then decreasing evenly to zero at the centre.

For lattice bracing the stress in the braces can be found by revolving Q in the direction of the brace and the column axis.

Calling the distance from centre to centre of tie plates "*a*," then the corresponding moment is *Qa*, and, with "*h*" as before, the force on the end of the tie plate equals $\frac{Qa}{h}$, from which the riveting can be calculated. All tie plate connections to column members should have at least two rivets. Recent tests of Emperger with 4 angle bars ($\left[\!\!\left[\right.\right.$) united with tie plates with one rivet in each end showed no more resistance than 4 separate angle bars, a result that was to be expected.

Cast Iron. Central Load.

Based upon the numerous tests by Prof. Tetmajer the following formula has been deduced by Prof. Ostenfeld.

$$S = \frac{7{\cdot}76}{1 + 0{\cdot}0007 \left(\frac{l}{r}\right)^2} \text{ t/cm}^2 \quad . \quad . \quad (20)$$

for which values are given in Table II., page 33.

Using *z* of formula (15) gives for determination of the dimension of a C.I. column to carry a given load based on above Rankine formula as developed by Prof. Claxton Fidler

$$A = A_0 \left(\tfrac{1}{2} + \sqrt{\tfrac{1}{4} + \frac{0{\cdot}0007\ z\ l^2}{A_0}}\right)$$

where $A_0 = \frac{P}{s}$ might be calculated allowing a factor of safety 10 for $s = 0{\cdot}7$ t/cm² = 4·4 tons/□″ = 10 000 lbs./□″.

It will be seen that, even if Rankine's formula were as safe for mild steel and wrought iron as J. B. Johnson's, it would give much more work to find the dimensions of a column than by using Prof. Ostenfeld's formula (16) based on Johnson's formula.

The radius of gyration *r* of a hollow round column with outside diameter *d* and metal *t* can be quickly found by

means of the following table, which goes beyond the limits used in practice.

$\frac{t}{d}$	$\frac{r}{d}$
0·000	0·353
0·020	0·346
0·040	0·339
0·060	0·333
0·080	0·326
0·100	0·320
0·120	0·314
0·140	0·308
0·160	0·302
0·180	0·297
0·200	0·292
0·220	0·286
0·240	0·281
0·260	0·277
0·280	0·273
0·300	0·269

Ex.—A 12″ column has $1\frac{1}{4}$″ metal, find radius of gyration :—

$$\frac{t}{d} = \frac{1\frac{1}{4}}{12} = 0{\cdot}104.$$

Interpolation :—

$$\frac{4}{20} = \frac{x}{6}$$

$$x = \frac{4 \times 6}{20} = 1$$

$$\frac{r}{d} = 0{\cdot}320 - 0{\cdot}001 = 0{\cdot}319$$

hence for $d = 12''$

$$r = 0{\cdot}319 \times 12 = 3{\cdot}83''.$$

Comparison of Formulæ for Mild Steel and Wrought Iron with Prof. Tetmajer's Tests.

The accompanying diagrams, Figs. 1 to 3, have been photographically reproduced, Fig. 1 from Prof. Ostenfeld's and Figs. 2 and 3 from Prof. Tetmajer's applied mechanics, and the author has added some curves (see page 21).

Figs. 1 and 2 show that the curve of Johnson-Ostenfeld and Euler agree well with the means of the tests. A heavy dot indicates the centre of a group of tests, and is usually close to the curve. The group centre for the shortest length in both cases was produced (exceptionally) by flat ended struts, taking $l = \frac{L}{2}$, hence they do not count much. The results of the mentioned formula can therefore be used with confidence for designing and checking.

For cast-iron, Fig. 3 shows good agreement with Rankine-Ostenfeld.

The illustrations also show Tetmajer's "straight lines" for

Mild steel . $$S = 3{\cdot}10 - 0{\cdot}0114\frac{l}{r}\ \text{t/cm}^2 \quad (21)$$

$$10 \angle \frac{l}{r} \angle 105$$

Wrought iron . $$S = 3{\cdot}03 - 0{\cdot}0129\frac{l}{r}\ \text{t/cm}^2 \quad (22)$$

$$10 \angle \frac{l}{r} \angle 112$$

and also his parabola for

Cast iron $$S = 7{\cdot}76 - 0.120\frac{l}{r} + 0{\cdot}00053\left(\frac{l}{r}\right)^2\ \text{t/cm}^2 \quad (23)$$

$$5 \angle \frac{l}{r} \angle 80$$

continued by his Euler, for E = 1000 t/cm².

Further, the results of Prof. Claxton Fidler's formula, from his Theory and Practice of Bridge Construction, have been plotted on the diagrams; these were not intended for mean values but for safe values as the diagrams also show them to be, except for short columns of mild steel.

Plotting Johnson-Ostenfeld and Euler on Hodgkinson's tests in Prof. Fidler's book, shows good agreement for wrought iron, equally Rankine-Ostenfeld for cast iron.

Diagram Fig. 1 for mild steel shows the results of a Rankine formula

$$S = \frac{3{\cdot}27}{1 + 0{\cdot}000093\left(\frac{l}{r}\right)^2}\ \text{t/cm}^2 \quad . \quad . \quad (24)$$

which is determined by the method of the minor squares from Prof. Tetmajer's tests by Prof. Ostenfeld, and, hence should be most reliable, but as Prof. Ostenfeld remarks, Rankine's formula does not agree well with the tests in general, and ought only to be used within certain limits for mild steel. It will be

seen that for long columns it shows higher values than can be obtained by tests, and is therefore dangerous to use. Other versions of Rankine's formula that agree well for short lengths are still worse for long lengths of mild steel.

Fig. 4 gives an interesting comparison of generally used methods, showing the great variation of ordinary practice and the importance of using formulæ that are based upon tests.

Wood. Central Load.

Prof. Tetmajer has made numerous tests of **Pine** columns with steel pointed ends : from these he deduces the "straight line" formula

$$S = 0{\cdot}293 - 0{\cdot}0194\,\frac{l}{r} \quad \text{t/cm}^2 \qquad . \quad (25)$$

$$5 < \frac{l}{r} < 100$$

continued by Euler with $E = 100$ t/cm^2.

Table II., page 33, shows the results of his formulæ, the metric values being taken from his book.

However, it often happens that other timbers than European pine are used for columns, and the engineer may then feel at a loss, but the difficulty can be got over as mentioned when dealing with J. B. Johnson's formula. As an example of making such formula shall be taken **Pitch Pine** using the mean values given by Rivington namely $B = 6400$ lbs./□" and $E = 1\,500\,000$ lbs./□".

Formula (8) gives

$$C = \frac{B^2}{4\pi^2 E} = \frac{6400^2}{39{\cdot}48 \times 1\,500\,000} = 0{\cdot}69$$

and (7) gives the limit

$$\frac{l}{r} = \sqrt{\frac{2\pi^2 E}{B}} = \sqrt{\frac{19{\cdot}74 \times 1\,500\,000}{6400}} = 68.$$

Hence by (4) breaking load by J. B. Johnson.

$$S = B - C\left(\frac{l}{r}\right)^2 = 6400 - 0{\cdot}69\left(\frac{l}{r}\right)^2 \text{ lbs./□" up to } \frac{l}{r} = 68.$$

Above this ratio must be worked by Euler (3)

$$S = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2} = \frac{9{\cdot}87 \times 1500\,000}{\left(\frac{l}{r}\right)^2} = \frac{14\,800\,000}{\left(\frac{l}{r}\right)^2} \text{ lbs./}\square''$$

as breaking load for pitch pine columns with free ends.

Eccentrically Loaded Columns.

The following formula of Prof. Ostenfeld gives reliable results for the breaking load of an eccentrically loaded column with free ends

$$\mathbf{S = \frac{B}{I + \frac{f}{k} + c\left(\frac{l}{r}\right)^2}} \quad . \quad . \quad (26)$$

where f is the eccentricity, k the radius of core and B and c are constants given below. It will be noted that B which is the breaking load of a short bar, has somewhat different values than those given for central loads, thus however agreeing better with tests for eccentric loads.

Mild steel . $B = 3{\cdot}36$ t/cm^2 $= 21{\cdot}3$ tons/$\square''$ $= 47\,800$ lbs./$\square''$, $c = 0{\cdot}00018$.

Wrought iron. $B = 3{\cdot}03$ t/cm^2 $= 19{\cdot}2$ tons/$\square''$ $= 43\,000$ lbs./$\square''$, $c = 0{\cdot}00018$.

Cast iron . $B = 7{\cdot}00$ t/cm^2 $= 44{\cdot}4$ tons/$\square''$ $= 99\,600$ lbs./$\square''$, $c = 0{\cdot}00085$.

Pine or oak . $B = 0{\cdot}35$ t/cm^2 $= 2{\cdot}2$ tons/$\square''$ $= 5000$ lbs./$\square''$, $c = 0{\cdot}00035$.

For use in designing he has developed

$$\mathbf{A = A_0\left[\tfrac{1}{2} + \sqrt{\tfrac{1}{4} + \frac{z}{A_0}(fe + cl^2)}\right]} \quad . \quad (27)$$

A, A_0 and z have the same meaning as in formula (16) for central load, and e is the distance from neutral axis through centre of gravity to the extreme fibre. To determine A find A_0 and z, as shown previously, and assume e (by a guess). When A has been found by the first trial, and a section is

selected, the corrected values of z and e are taken from this section and a fresh trial is made and then if necessary a further trial.

The development of the formula is worked out by Prof. Ostenfeld, but it is too long to reproduce here. He proves by Prof. Tetmajer's 9 tests for mild steel and 31 tests for wrought iron, that if the eccentricity is not too small the agreement with the tests, from which he deduced B and c, is good. Further, that the agreement is still good when $f = k$, while for smaller eccentricities the breaking weights obtained by (26) are somewhat too small, i.e. to the safe side anyhow.

He further mentions that for $f = 0$, the formula reduces itself to Rankine's formula, but that it will be seen that c has nearly double the value. For cast iron and wood no tests were made; he therefore prepared the values of B and c by analogies.

Prof. Ostenfeld mentions that a similar formula, published somewhat earlier by Dr. F. v. Emperger, was unknown to him when he developed his formula, and he calls attention to the fact that Emperger has not noticed that a greater value of c is necessary for the eccentric formula than for Rankine's to make agreement with the tests.

Ex. 8.—A column formed of two braced B.S. channel bars, $6'' \times 3\frac{1}{2}'' \times 17{\cdot}9$ lbs., is loaded with a 2″ eccentricity in the axial plane parallel to the webs. Free length $l = 153''$. Find safe load for a factor of safety 4.

$B = 21{\cdot}3$ tons/□″.

$k = \frac{R}{A} = \frac{9{\cdot}88}{5{\cdot}27} = 1''{\cdot}87$. (R and A are taken for one channel only.)

$\frac{f}{k} = \frac{2}{1{\cdot}87} = 1{\cdot}07.$

$r = 2{\cdot}36''$ (max. r for a channel).

$\frac{l}{r} = \frac{153}{2.36} = 65.$

$c\left(\frac{l}{r}\right)^2 = 0{\cdot}00018 \times 65^2 = 0.76.$

By (26): $S = \dfrac{B}{1 + \frac{f}{k} + c\left(\frac{l}{r}\right)^2} = \dfrac{21{\cdot}3}{1 + 1{\cdot}07 + 0{\cdot}76}$

$= \dfrac{21{\cdot}3}{2{\cdot}83} = 7{\cdot}53$ tons/□″.

(If the load had been central, Table I. shows that the breaking load would have been 15·0 tons/□″ for $\frac{l}{r} = 65$.)

$A = 2 \times 2{\cdot}57 = 10{\cdot}5$□″.

As the breaking load = A S

the safe load $= \dfrac{10{\cdot}5 \times 7{\cdot}53}{4} = 19{\cdot}8$ tons.

Ex. 9.—Find the section to be used in a column, formed of two braced channels, that will carry a load of 24 tons 2″ out of centre in a plane parallel to the webs. The length is 18′ 3″, the base is well bedded, and the top carries a ceiling.

Allow a stress of $s_0 = 5$ tons/□″; then factor of safety $= \dfrac{21{\cdot}3}{5} = 4{\cdot}26$.

$A_0 = \dfrac{P}{s_0} = \dfrac{24}{5} = 4{\cdot}8$□″.

Allow $z = 1{\cdot}0$ (see table of z values, page 13).

$\dfrac{z}{A_0} = \dfrac{1{\cdot}0}{4{\cdot}8} = 0{\cdot}21$.

Assume a 10″ channel with $e = 5$″.

$f e = 2 \times 5 = 10$.

$L = 18' 3'' = 219''$.

$l = 0{\cdot}7 \times 219 = 153''$; (foot held and top guided).

$c l^2 = 0{\cdot}00018 \times 153^2 = 4{\cdot}2$.

By (27): $A = A_0\left[\frac{1}{2} + \sqrt{\frac{1}{4} + \frac{z}{A_0}(f e + c l^2)}\right]$

$= 4{\cdot}8\left[0{\cdot}5 + \sqrt{0{\cdot}25 + 0{\cdot}21\,(10 + 4{\cdot}2)}\right]$

$= 4{\cdot}8\left[0{\cdot}5 + \sqrt{0{\cdot}25 + 2{\cdot}98}\right] = 4{\cdot}8\,(0{\cdot}5 + 1{\cdot}80) = 11{\cdot}0$□″.

For each channel is required $\frac{1}{2} \times 11{\cdot}0 = 5{\cdot}5\square''$. Try B.S. channel $7'' \times 3\frac{1}{2}'' \times 20{\cdot}2$ lb. with $5{\cdot}95\square''$. For this section we have

$$z = \frac{A^2}{I} = \frac{(2 \times 5{\cdot}95)^2}{2 \times 44{\cdot}5} = 1{\cdot}59$$

$$\frac{z}{A_0} = \frac{1{\cdot}59}{4{\cdot}8} = 0{\cdot}331.$$

$$e = \tfrac{1}{2} \times 7'' = 3{\cdot}5''.$$

$f\,e = 2 \times 3{\cdot}5 = 7.$

$c\,l^2 = 4{\cdot}2$ as before.

$$A = A_0 \left[\tfrac{1}{2} + \sqrt{\tfrac{1}{4} + \frac{z}{A_0}(f\,e + c\,l^2)} \right]$$

$$= 4{\cdot}8 \left[0{\cdot}5 + \sqrt{0{\cdot}25 + 0{\cdot}331\,(7 + 4{\cdot}2)} \right]$$

$$= 4{\cdot}8\,(0{\cdot}5 + \sqrt{3{\cdot}95}) = 4{\cdot}8 \times 2{\cdot}45 = 11{\cdot}7\square''$$

required, hence the column is safe as there are two channels with $2 \times 5{\cdot}95 = 11{\cdot}9\square''$.

Ex. 10.—A column formed of two German standard channels has a free length of 385 cm., and is required to carry 14·7 t. with an eccentricity of 10 cm. in axial plane parallel to webs. Factor of safety $4\frac{1}{2}$.

$B = 3{\cdot}36$ t/cm^2.

$$s_0 = \frac{3{\cdot}36}{4{\cdot}5} = 0{\cdot}75 \text{ t/cm}^2.$$

$$A_0 = \frac{P}{s_0} = \frac{14{\cdot}7}{0{\cdot}75} = 19{\cdot}6 \text{ cm}^2.$$

Assume two channels N.P. No. 24; as the height is 24 cm. we have

$e = \frac{1}{2} \times 24 = 12$ cm.

$f\,e = 10 \times 12 = 120.$

$c\,l^2 = 0{\cdot}00018 \times 385^2 = 27.$

Assume $z = 1{\cdot}2$, (see table of z values).

$$\tfrac{1}{4} + \frac{z}{A_0}(f\,e + c\,l^2) = \tfrac{1}{4} + \frac{1{\cdot}2}{19{\cdot}6}(120 + 27)$$

$$= 0{\cdot}25 + 9\ 00 = 9{\cdot}25.$$

$$A = A_0\left[\tfrac{1}{2} + \sqrt{\tfrac{1}{4} + \frac{z}{A_0}(f\,e + c\,l^2)}\right]$$

$$= A_0\,(\tfrac{1}{2} + \sqrt{9{\cdot}25}) = 19{\cdot}6\,(0{\cdot}5 + 3{\cdot}04) = 69.4\ \text{cm}^2.$$

the area of channels assumed $= 2 \times 42{\cdot}3 = 84{\cdot}6$ cm^2, therefore try a smaller channel, say No. 20; area $A = 2 \times 32{\cdot}2 = 64{\cdot}4$ cm^2.

$$z = \frac{A^2}{I} = \frac{64{\cdot}4^2}{2 \times 1910} = 1{\cdot}08.$$

$$e = \tfrac{1}{2} \times 20 = 10\ \text{cm.}$$

$$f\,e = 10 \times 10 = 100.$$

$$\sqrt{\tfrac{1}{4} + \frac{z}{A_0}(f\,e + c\,l^2)} = \sqrt{\tfrac{1}{4} + \frac{1{\cdot}08}{19{\cdot}6}(100 + 27)}$$

$$= \sqrt{0{\cdot}25 + 7{\cdot}00} = \sqrt{7{\cdot}25} = 2{\cdot}70.$$

$$A = A_0\left[\tfrac{1}{2} + \sqrt{\tfrac{1}{4} + \frac{z}{A_0}(f\,e + c\,l^2)}\right]$$

$$= 19{\cdot}6\,(0{\cdot}5 + 2{\cdot}70) = 62{\cdot}6\ \text{cm}^2.$$

required, and the area of the channels assumed equals 64·4 cm^2., therefore the two channels N.P. No. 20 can be used.

The actual factor of safety for the two No. 20 is found as follows:—

$$k = \frac{R}{A} = \frac{191}{32{\cdot}2} = 5{\cdot}9\ \text{cm. (found by one channel).}$$

$$\frac{f}{k} = \frac{10}{5{\cdot}9} = 1{\cdot}70.$$

$$\frac{l}{r} = \frac{385}{7{\cdot}71} = 50.$$

$$c\left(\frac{l}{r}\right)^2 = 0{\cdot}00018 \times 50^2 = 0{\cdot}45.$$

$$S = \frac{B}{1 + \frac{f}{k} + c\left(\frac{l}{r}\right)^2} = \frac{3{\cdot}36}{1 + 1{\cdot}70 + 0{\cdot}45} = 1{\cdot}07 \text{ t/cm}^2.$$

Breaking load $A\,S = 64{\cdot}4 \times 1{\cdot}07 = 69$ t.

Factor of safety $\frac{69}{14{\cdot}7} = 4{\cdot}7$ or fully the $4\frac{1}{2}$ required.

If in any case the eccentricity f is greater than the radius of core k there will be tension in part of the metal, and as **Cast Iron** has a small tensile strength the amount of tension should be considered. Prof. Ostenfeld has prepared the following formula for the tensile stress

$$s^1 = s\left(\frac{0{\cdot}85\ S^1 f}{k\,(0{\cdot}85\ S^1 - s)} - 1\right) \quad . \quad . \quad (28)$$

s^1 is the max. unitary tensile stress.

$s = \frac{P}{A}$ is the stress evenly distributed over the area.

$S^1 = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2} = \frac{10\ E}{\left(\frac{l}{r}\right)^2}$ is the breaking stress by Euler's formula (3).

For C.I. say $E = 1000$ t/cm^2 $= 6300$ tons/□″ $= 14\,000\,000$ lbs./□″.

0·85 is an empirical constant which Prof. Ostenfeld deduced from Tetmajer's test for mild steel and wrought iron (formula (28) holds good for any material) and is so near his approximate theoretical value $\frac{8}{\pi^2} = 0{\cdot}81$ that the formula (28) can be used with confidence.

In case of a great eccentricity the resistance of C.I. column depends entirely upon the tensile stress, and the resistance will in that case not be expressed by (26) but be governed by above formula (28).

The resistance will then be $A\,s$ where s is found by (28), which can be transcribed to

$$s^2 + \left[s^1 + 0{\cdot}85\,S^1\left(\frac{f}{k} - 1\right)\right]s - 0{\cdot}85\,S^1 s^1 = 0 \quad (29)$$

Having worked up the numerical values, we find s out of a plain quadratic equation (29).

Prof. Ostenfeld allows for C.I. as working tensile stress: $s^1 = 0{\cdot}2$ to $0{\cdot}3$ t/cm^2 = $1\frac{1}{4}$ to $1{\cdot}9$ tons/□″ · = 2800 to 4300 lbs./□″.

Ex. 11.—A 12″ diam. C.I. column with 1″ metal and l = 12′ 6″ free length is loaded with 3″ eccentricity; find the safe load for factor of safety 10, tensile stress not to exceed $1\frac{1}{2}$ tons/□″.

$\frac{t}{d} = \frac{1''}{12''} = 0{\cdot}083$ and by table for rad. of gyr. (see heading Cast Iron, page 19) $r = 0{\cdot}325 \times 12'' = 3{\cdot}90''$.

$$k = \frac{r^2}{e} = \frac{3{\cdot}90^2}{6} = 2{\cdot}53''.$$

Formula (26) will be used.

B = 44·4 tons/□″.

$$\frac{f}{k} = \frac{3}{2{\cdot}53} = 1{\cdot}18.$$

$$\frac{l}{r} = \frac{150}{3{\cdot}90} = 38{\cdot}4.$$

$$c\left(\frac{l}{r}\right)^2 = 0{\cdot}00085 \times 38{\cdot}4^2 = 1{\cdot}25.$$

$$S = \frac{B}{1 + \frac{f}{k} + c\left(\frac{l}{r}\right)^2} = \frac{44{\cdot}4}{1 + 1{\cdot}18 + 1{\cdot}25} = \frac{44{\cdot}4}{3{\cdot}43} = 12{\cdot}9$$

tons/□″.

$$A = \frac{\pi}{4}12^2 - \frac{\pi}{4}10^2 = 113{\cdot}1 - 78{\cdot}5 = 34{\cdot}6\text{□″}.$$

Breaking load A S = $34{\cdot}6 \times 12{\cdot}9 = 446$ tons.

Safe load $\frac{446}{10} = 44{\cdot}6$ tons.

However, as the eccentricity $f = 3''$ is greater than the core radius $k = 2{\cdot}53''$, the tensile stress should be examined by formula (28).

$$s = \frac{P}{A} = \frac{44{\cdot}6}{34{\cdot}6} = 1{\cdot}29 \text{ tons/□″}.$$

$$S^1 = \frac{10\,E}{\left(\frac{l}{r}\right)^2} = \frac{10 \times 6300}{38{\cdot}4^2} = 42{\cdot}7 \text{ tons}/\square''.$$

$$0{\cdot}85\ S^1 = 0{\cdot}85 \times 42{\cdot}7 = 36{\cdot}3.$$

$$0{\cdot}85\ S^1 - s = 36{\cdot}3 - 1{\cdot}29 = 35{\cdot}0.$$

$$\frac{0{\cdot}85\ S^1 f}{k\,(0{\cdot}85\ S^1 - s)} = \frac{36{\cdot}3 \times 3}{2{\cdot}53 \times 35{\cdot}0} = 1{\cdot}23$$

Tensile stress $s^1 = s\left(\frac{0{\cdot}85\ S^1 f}{k\,(0{\cdot}85\ S^1 - s)} - \right) = 1{\cdot}29\,(1{\cdot}23 - 1)$

$= 1{\cdot}29 \times 0{\cdot}23 = 0{\cdot}29$ tons/$\square''$: hence safe.

If the load had been acting on a bracket, so that $f = 10''$, all would depend upon the tensile stress, and s must be found by formula (29).

$s^1 = 1{\cdot}5$ tons/$\square''$ is allowed.

$$\frac{f}{k} = \frac{10}{2{\cdot}53} = 3{\cdot}95.$$

$0{\cdot}85\ S^1 = 36{\cdot}3$ as above.

$$0{\cdot}85\ S^1\left(\frac{f}{k} - 1\right) = 36{\cdot}3\,(3{\cdot}95 - 1) = 36{\cdot}3 \times 2{\cdot}95 = 107{\cdot}1.$$

$$s^1 + 0{\cdot}85\ S^1\left(\frac{f}{k} - 1\right) = 1{\cdot}5 + 107{\cdot}1 = 108{\cdot}6.$$

$$0{\cdot}85\ S^1 s^1 = 36{\cdot}3 \times 1{\cdot}5 = 54{\cdot}4.$$

hence (29) $s^2 + \left[s^1 + 0{\cdot}85\ S^1\left(\frac{f}{k} - 1\right)\right] s - 0{\cdot}85\ S^1 s^1 = 0$

gives $s^2 + 108{\cdot}6\,s - 54{\cdot}4 = 0.$

For the quadratic equation

$$x^2 + a\,x + c = 0$$

the solution is

$$x = -\frac{a}{2} \pm \sqrt{\left(\frac{a}{2}\right)^2 - c}$$

and in this case minus in front of root cannot be used, as the result must be positive.

Hence the quadratic equation for s gives the safe evenly distributed stress

$$s = -54{\cdot}3 + \sqrt{54{\cdot}3^2 + 54{\cdot}4} = -54{\cdot}3 + \sqrt{3003}$$
$$= -54{\cdot}3 + 54{\cdot}30 = 0{\cdot}50 \text{ tons/}\square''.$$

Safe load $A\,s = 34{\cdot}6 \times 0{\cdot}50 = 17$ tons.

If this case of 10″ eccentricity had been worked out by (26) the breaking load would be found to be $7{\cdot}16$ tons/□″. By factor of safety 10 the safe load should be $0{\cdot}72$ tons/□″, while as just found only $0{\cdot}50$ tons/□″ would be permissible.

A Column affected by a Bending Moment.

Prof. Ostenfeld proves that a reasonable result is obtained when a column is affected by an axial load P and a bending moment M when we calculate as if the column was loaded with P, having the eccentricity $f = \frac{M}{P}$.

He works out the following example.

A square timber of 4 m. length, is supported at the ends and carries a normal load of $1{\cdot}4$ t. per metre as well as an axial load of $8{\cdot}4$ t. Find the scantling when a stress of 60 kg/cm² is allowed.

$$M = \frac{1{\cdot}4 \times 4^2}{8} = 2{\cdot}8 \text{ m.t. (metre-tons).}$$

$$P = 8{\cdot}4 \text{ t.}$$

$$f = \frac{M}{P} = \frac{2{\cdot}8}{8{\cdot}4} = 0{\cdot}33 \text{ m.} = 33 \text{ cm.}$$

Stress $s_0 = 60$ kg/cm² $= 0{\cdot}06$ t/cm².

$$A_0 = \frac{P}{s_0} = \frac{8{\cdot}4}{0{\cdot}06} = 140 \text{ cm}^2.$$

$z = 12$ for a square section.

Assume a 34 cm. timber, or say $e = 17$ cm.

$fe = 33 \times 17 = 561.$

$l = 4$ m. $= 400$ cm.

$c l^2 = 0{\cdot}00035 \times 400^2 = 56.$

$$\frac{z}{A_0}(f e + c l^2) = \frac{12}{140}(561 + 56) = \frac{12 \times 617}{140} = 52{\cdot}8.$$

Formula (27) gives

$$A = A_0 \left[\tfrac{1}{2} + \sqrt{\tfrac{1}{4} + \frac{z}{A_0}(f e + c l^2)}\right]$$

$$= 140\,(0{\cdot}5 + \sqrt{0{\cdot}25 + 52{\cdot}8}) = 140\,(0{\cdot}5 + \sqrt{53{\cdot}0})$$

$$= 140\,(0{\cdot}5 + 7{\cdot}28) = 140 \times 7{\cdot}78 = 1090 \text{ cm}^2.$$

Hence the baulk should be $\sqrt{1090} = 33$ cm.

There is no reason to repeat the calculation, as the difference between $e = \frac{1}{2} \times 33 = 16{\cdot}5$ cm. and the assumed value 17 cm. is so very small.

TABLE I.

BREAKING LOAD AND STRESS FACTOR FOR MILD STEEL AND WROUGHT IRON. CENTRALLY LOADED COLUMNS WITH "FREE" ENDS.

$\frac{l}{r}$	Mild Steel					Stress Factor for Steel or Wrought Iron		Wrought Iron				$\frac{l}{r}$
	$t/cm.^2$	Diff.	Tons/□"	Diff.	Lbs./□"		Diff.	Lbs./□"	Tons/□"	t/cm^2	Diff.	
0	2·72	0	17·3	0	38 700	1·000	1	36 700	16·4	0 2·58	0	0
5	2·72	1	17·3	1	38 700	0·999	2	36 700	16·4	1 2·58	1	5
10	2·71	1	17·2	0	38 500	0·997	4	36 600	16·3	0 2·57	1	10
15	2·70	1	17·2	1	38 400	0·993	6	36 500	16·3	1 2·56	1	15
20	2·69	2	17·1	2	38 300	0·987	8	36 300	16·2	1 2·55	2	20
25	2·67	2	16·9	1	38 000	0·979	9	36 000	16·1	2 2·53	2	25
30	2·65	3	16·8	2	37 700	0 970	11	35 700	15·9	2 2·51	3	30
35	2·62	4	16·6	2	37 300	0·959	12	35 300	15·7	2 2·48	3	35
40	2·58	3	16·4	2	36 800	0·947	14	34 900	15·5	2 2·45	4	40
45	2·55	4	16·2	3	36 300	0 933	16	34 300	15·3	2 2·41	4	45
50	2·51	5	15·9	3	35 700	0·917	18	33 700	15·1	3 2·37	4	50
55	2·46	5	15·6	3	35 000	0·899	19	33 100	14·8	3 2·33	5	55
60	2·41	5	15·3	3	34 300	0·880	21	32 400	14·5	4 2·28	6	60
65	2·36	6	15·0	4	33 600	0·859	22	31 600	14·1	3 2·22	5	65
70	2·30	7	14·6	4	32 700	0·837	24	30 900	13·8	4 2·17	6	70
75	2·23	7	14·2	5	31 700	0·813	26	30 000	13·4	4 2·11	7	75
80	2·16	7	13·7	4	30 700	0·787	28	29 000	13 0	5 2·04	7	80
85	2·09	7	13·3	5	29 800	0·759	29	28 000	12·5	5 1 97	7	85
90	2·02	9	12·8	5	28 700	0·730	31	27 000	12·0	4 1·90	8	90
95	1·93	8	12·3	6	27 500	0·699	32	25 900	11·6	6 1·82	8	95
100	1·85	9	11·7	5	26 300	0·667	34	24 700	11·0	5 1·74	9	100
105	1·76	9	11·2	6	25 000	0·633	36	23 500	10·48	58 1·65	9	105
110	1·67	10	10·58	63	23 800	0·597	38	22 200	9·90	59 1·56	9	110
115	1·57	11	9 95	65	22 300	0·559	39	20 900	9·31	64 1·47	10	115
120	1·46	10	9·30	68	20 800	0·520	40	19 500	8·67	65 1·37	11	120
125	1·36	10	8·62	64	19 300	0·480	36	18 000	8·02	60 1·26	9	125
130	1·26	10	7·98	58	17 900	0·444	32	16 600	7·42	54 1·17	9	130
135	1·16	8	7·40	52	16 500	0·412	29	15 400	6·88	59 1·08	7	135
140	1·08	7	6·88	48	15 400	0·383	26	14 300	6·39	43 1·007	68	140
145	1·009	66	6·40	41	14 400	0·357	26	13 400	5·96	39 0·939	62	145
150	0·943	60	5·99	39	13 400	0·333	21	12 500	5·57	35 0·877	55	150
155	0·883	54	5·60	34	12 600	0·312	19	11 700	5·22	33 0·822	51	155
160	0·829	50	5·26	31	11 800	0·293	18	11 000	4·89	29 0·771	46	160
165	0·779	45	4·95	29	11 100	0·275	15	10 300	4·60	26 0·725	42	165
170	0·734	41	4·66	26	10 450	0·260	15	9 720	4·34	24 0·683	38	170
175	0·693	38	4·40	24	9 860	0·245	14	9 170	4·10	23 0·645	36	175
180	0·655	35	4·16	22	9 320	0·231	12	8 660	3·87	21 0·609	32	180
185	0·620	32	3·94	21	8 820	0·219	11	8 200	3·66	19 0·577	30	185
190	0·588	30	3·73	19	8 360	0·208	11	7 770	3·47	18 0·547	28	190
195	0·558	27	3·54	17	7 940	0·197	10	7 380	3·29	16 0·519	25	195
200	0·531	..	3·37	..	7 550	0·187	..	7 030	3·13	.. 0·494	..	200

TABLE II.

BREAKING LOAD OF CAST-IRON AND EUROPEAN PINE. CENTRALLY LOADED COLUMNS WITH "FREE" ENDS.

$\frac{l}{r}$	Cast Iron					European Pine				$\frac{l}{r}$
	t/cm^2	Diff.	Tons/□"	Diff.	Lbs./□"	Lbs./□"	Tons/□"	Diff.	t/cm^2 Diff.	
10	7·25	55	46·0	35	103 100	3 900	1·74	6	0 274 10	10
15	6·70	64	42·5	40	95 300	3 750	1·68	7	0·264 10	15
20	6·06	66	38·5	42	86 200	3 610	1·61	6	0·254 10	20
25	5·40	64	34·3	41	76 800	3 470	1·55	6	0·244 9	25
30	4·76	58	30·2	37	67 700	3 340	1·49	6	0·235 10	30
35	4·18	52	26·5	33	59 400	3 200	1·43	6	0·225 9	35
40	3·66	45	23·2	28	52 100	3 070	1·37	6	0·216 10	40
45	3·21	39	20·4	25	45 700	2 930	1·31	6	0·206 10	45
50	2·82	33	17·9	21	40 100	2 790	1·25	7	0·196 10	50
55	2·49	29	15·8	18	35 400	2 640	1·18	6	0·186 9	55
60	2·20	24	14·0	16	31 300	2 510	1·12	6	0·177 10	60
65	1·96	21	12·4	13	27 900	2 370	1·06	6	0·167 10	65
70	1·75	18	11·11	114	24 900	2 230	1·00	7	0·157 10	70
75	1·57	15	9·97	95	22 400	2 090	0·93	5	0·147 9	75
80	1·42	14	9·02	89	20 200	1 960	0·88	7	0·138 10	80
85	1·28	12	8·13	77	18 200	1 820	0·81	6	0·128 10	85
90	1·16	10	7·36	63	16 500	1 680	0·75	6	0·118 9	90
95	1·061	91	6·73	57	15 100	1 550	0·69	6	0·109 10	95
100	0·970	79	6·16	50	13 800	1 410	0·63	6	0·099 9	100
105	0·891	71	5·66	45	12 700	1 280	0·57	5	0·090 8	105
110	0·820	63	5·21	41	11 700	1 170	0·52	4	0·082 7	110
115	0·757	57	4·80	36	10 770	1 070	0·48	4	0·075 6	115
120	0·700	50	4·44	31	9 960	980	0·44	4	0·069 6	120
125	0·650	45	4·13	29	9 240	900	0·40	3	0·063 5	125
130	0·605	42	3·84	27	8 600	830	0·37	3	0·058 4	130
135	0·563	36	3·57	22	8 010	770	0·34	3	0·054 4	135
140	0·527	33	3·35	21	7 490	710	0·32	2	0·050 3	140
145	0·494	31	3·14	20	7 020	670	0·30	2	0·047 3	145
150	0·463	..	2·94	..	6 580	630	0·28	..	0·044 ..	150

RADII OF GYRATION FOR GERMAN SECTIONS.

Common H Beams	r max.	r min.	Broad H Beams	r max.	r min.	⊔ Bars	r max.	r min.	T Bars	r for Axis: Parallel to Table	r for Axis: Normal to Table
No.	*cm.*	*cm.*	No.	*cm.*	*cm.*	No.	*cm.*	*cm.*	No.	*cm.*	*cm.*
8	3·20	0·91	18B	7·64	4·24	8	3·11	1·33	8/4	0·99	1·90
9	3·61	0·99	20B	8·61	4·72	10	3·90	1·47	9/4½	1·12	2·13
10	4·01	1·07	22B	9·45	5·18	12	4·63	1·59	10/5	1·25	2·37
11	4·40	1·15	24B	10·3	5·6	14	5·44	1·75	12/6	1·49	2·84
12	4·80	1·23	25B	10·7	5·8	16	6·21	1·89	14/7	1·74	3·36
13	5·20	1·30	26B	11·1	6·0	18	6·96	2·02	16/8	1·99	3·78
14	5·61	1·39	27B	11·6	6·3	20	7·71	2·14	18/9	2·24	4·26
15	6·00	1·46	28B	12·0	6·6	22	8·50	2·30	20/10	2·47	4·69
16	6·40	1·55	29B	12·4	7·0	24	9·23	2·42			
17	6·80	1·63	30B	12·9	7·0	26	10·0	2·57	7/7	2·05	1·44
18	7·20	1·71	32B	13·7	7·0	28	10·8	2·74	8/8	2·33	1·65
19	7·60	1·79	34B	14·5	7·0	30	11·6	2·90	9/9	2·64	1·85
20	8·00	1·87	36B	15·3	7·0				10/10	2·93	2·05
21	8·40	1·94	38B	16·1	6·9				12/12	3·51	2·45
22	8·80	2·03	40B	16·8	6·9				14/14	4·07	2·88
23	9·20	2·10	42½B	17·8	6·9						
24	9·59	2·18	45B	18·8	6·8						
25	9·98	2·27	47½B	19·8	6·8						
26	10·4	2·32	50B	20·6	6·7						
27	10·8	2·39	55B	22·5	6·6						
28	11·1	2·44	60B	24·4	6·5						
29	11·5	2·50	65B	26·1	6·4						
30	11·9	2·55	70B	28·2	6·3						
32	12·7	2·67	75B	29·0	6·2						
34	13·4	2·79									
36	14·2	2·90									
38	15·0	3·01									
40	15·7	3·14									
42½	16·7	3·29									
45	17·7	3·43									
47½	18·6	3·58									
50	19·6	3·72									
55	21·6	4·06									
60	23·4	4·26									

Above are the Differdange (Differdinger) or Grey Broad Flange Beams

RADII OF GYRATION FOR GERMAN SECTIONS.

Equal Angles	Thickness	r max.	r min.	Two Back to Back r min.	Equal Angles	Thickness	r max.	r min.	Two Back to Back r min.	Tube of 4 Phoenix Bars	Thickness of Tube	r (constant)
No	*mm.*	*cm.*	*cm.*	*cm.*	No.	*mm.*	*cm.*	*cm.*	*cm.*	No.	*mm.*	*cm.*
4	4	1·55	0·79	1·23	9	9	3·50	1·77	2·77	5	4	4·41
,,	6	1·52	0·78	1·20	,,	11	3·46	1·76	2·75	,,	8	4·35
,,	8	1·46	0·77	1·17	,,	13	3·42	1·76	2·72			
										7½	6	6·14
4½	5	1·74	0·88	1·38	10	10	3·88	1·97	3·08	,,	10	6·10
,,	7	1·70	0·88	1·35	,,	12	3·85	1·96	3·05			
,,	9	1·64	0·86	1·31	,,	14	3·81	1·95	3·02	10	8	7·92
										,,	12	7·90
5	5	1·94	0·98	1·54	11	10	4·23	2·16	3·35			
,,	7	1·90	0·98	1·51	,,	12	4·21	2·15	3·34	12½	10	9·72
,,	9	1·85	0·97	1·46	,,	14	4·18	2·14	3·32	,,	14	9·66
5½	6	2·13	1·08	1·69	12	11	4·68	2·36	3·71	15	12	11·5
,,	8	2·09	1·07	1·65	,,	13	4·64	2·35	3·68	,,	18	11·5
,,	10	2·03	1·06	1·63	,,	15	4·60	2·35	3·65			
6	6	2·33	1·18	1·85	13	12	5·00	2·54	3·97			
,,	8	2·29	1·17	1·82	,,	14	4·97	2·53	3·94			
,,	10	2·22	1·15	1·77	,,	16	4·94	2·52	3·92			
6½	7	2·51	1·28	1·99	14	13	5·46	2·76	4·32			
,,	9	2·48	1·27	1·97	,,	15	5·42	2·75	4·30			
,,	11	2·41	1·26	1·92	,,	17	5·33	2·73	4·23			
7	7	2·72	1·38	2·15	15	14	5·77	2 93	4·57			
,,	9	2·68	1·37	2·13	,,	16	5·74	2·93	4·56			
,,	11	2·63	1·35	2·08	,,	18	5·71	2·93	4·54			
7½	8	2·90	1·47	2·30	16	15	6·23	3·15	4·94			
,,	10	2·87	1·47	2·28	,,	17	6·19	3·14	4·91			
,,	12	2·83	1·46	2·25	,,	19	6·10	3·12	4·84			
8	8	3·11	1·57	2·46								
,,	10	3·07	1·56	2·44								
,,	12	3·03	1·56	2·41								

The number of the German Standard indicates the principal dimension in centimetres.

APPENDIX.

Columns of Wood.

Some time after the treatise was written the Author obtained a copy of the results of Prof. Tetmajer's tests. The scantlings Tetmajer used for wood were large enough, being from 12 to 15 cms., and the tests were made with oak and four different classes of pine. Plotting the results showed that J. B. Johnson's Parabola gave too high results, while *T. H. Johnson's Straight Line* (a tangent from compressive stress B to the Euler curve) gave fair results.

The straight line from Tetmajer's mean value of B for oak fell in the mean of the tests, which only deviated very little. Similar results ought to be expected from leaf woods, or at all events from hard-woods.

The tests for pine woods were very erratic, and the straight line from Tetmajer's mean values of B fell above the mean of the tests, but a line from about 10 per cent. lower than the compressive stress gave a fair mean of the tests.

Hence when it is required to make column formulæ for a class of wood for which no column tests exist, Euler's formula should be used for the greater lengths and T. H. Johnson's for the shorter lengths. Use for leaf woods B equal to compressive stress, and for pine woods B about 10 per cent. less than this stress.

T. H. Johnson's formula is for $x = \frac{l}{r}$

$$S = B - Cx \quad . \quad . \quad . \quad . \quad . \quad (30)$$

with limit

$$\frac{l}{r} = x = \sqrt{\frac{3\pi^2 E}{B}} \quad . \quad . \quad (31) \quad 3\pi^2 = 29{\cdot}61$$

when x is calculated we obtain

$$C = \frac{2\pi^2 E}{x^2} \quad . \quad . \quad (32) \quad 2\pi^2 = 19{\cdot}74.$$

Ex. pitch pine with E = 1,500,000 lbs./□″ and compressive stress 6400 lbs./□″.

About 10 per cent. less than 6400 gives B to the safe side equal to 5700.

By (31) $\frac{l}{r} = x = \sqrt{\frac{3\pi^2 E}{B}} = \sqrt{\frac{29{\cdot}61 \times 1{,}500{,}000}{5700}} = 88{\cdot}3.$

By (32) $C = \frac{2\pi^2 E}{x^2} = \frac{19{\cdot}74 \times 1{,}500{,}000}{88{\cdot}3^2} = 43{\cdot}1.$

Hence by (30)

$$S = B - Cx = 5700 - 43{\cdot}1\,\frac{l}{r} \text{ lbs./□″ up to } \frac{l}{r} = 88.$$

Beyond this, Euler's formula as found by the same example calculated according to J. B. Johnson's formula.

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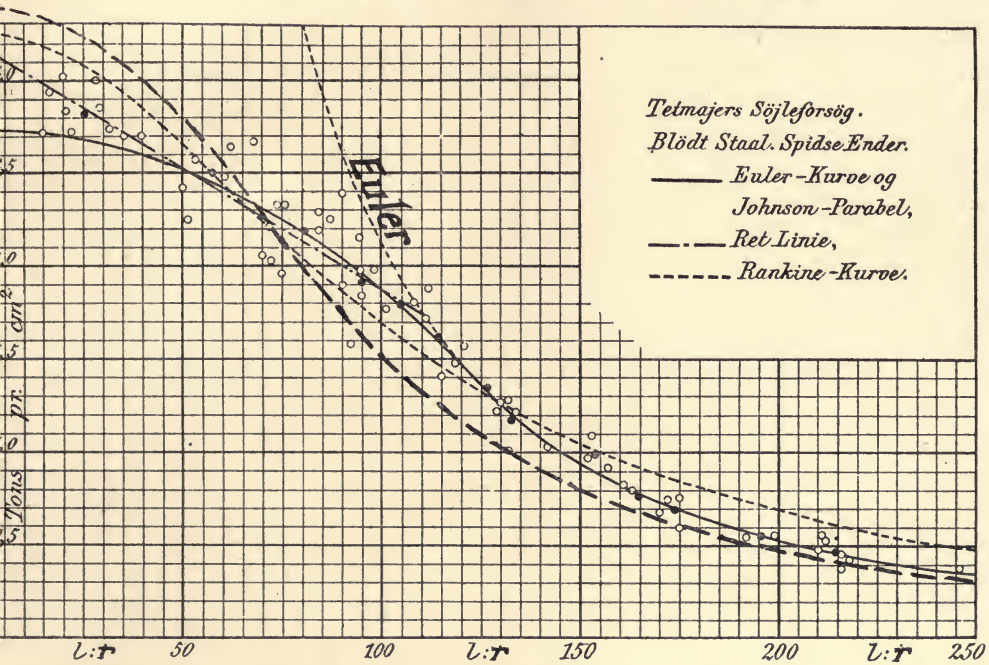
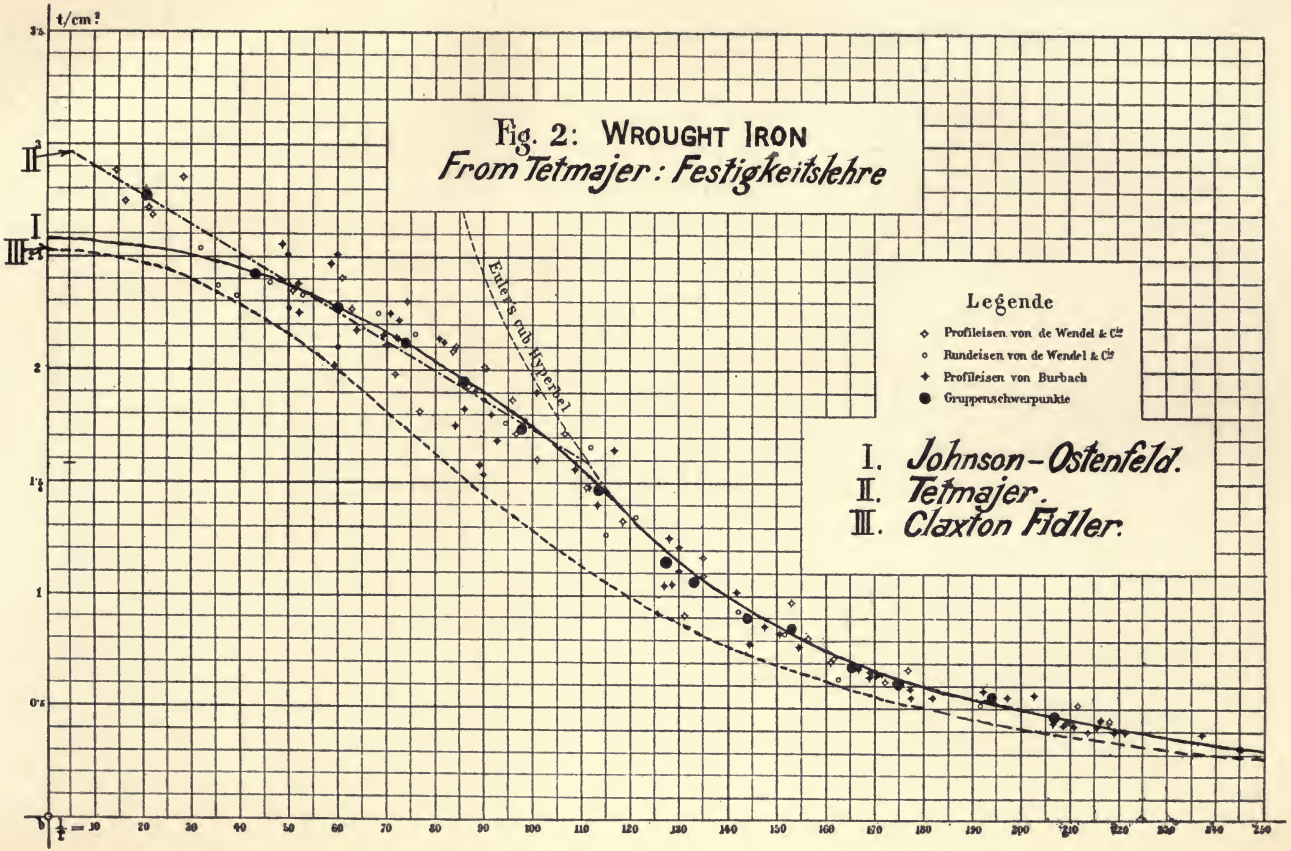
Fig. 1 : MILD STEEL.

From Ostenfeld : Elasticitetslære.



I. *Johnson – Ostenfeld.*
II. *Tetmajer.*
III. *Claxton Fidler.*
IV. *Rankine by Ostenfeld.*


t/cm²
Fig. 2: WROUGHT IRON
From Tetmajer: Festigkeitslehre
Euler's cub. Hyperbel
Legende
Profileisen von de Wendel & Cie
Rundeisen von de Wendel & Cie
Profileisen von Burbach
Gruppenschwerpunkte
I. Johnson-Ostenfeld.
II. Tetmajer.
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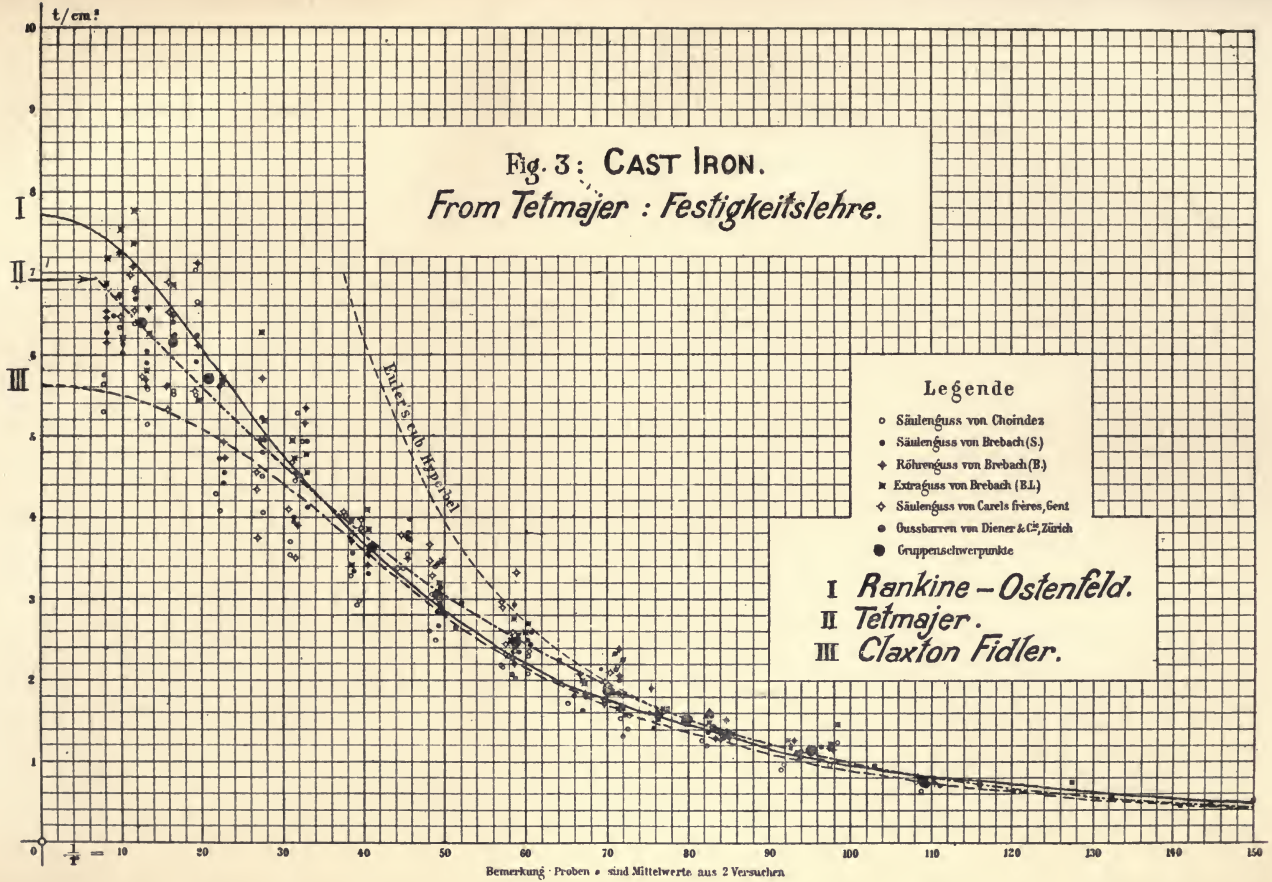
t/cm²
Fig. 3: CAST IRON.
From Tetmajer : Festigkeitslehre.
Euler's cub. Hyperbel
Legende
Säulenguss von Choindez
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Bemerkung: Proben • sind Mittelwerte aus 2 Versuchen

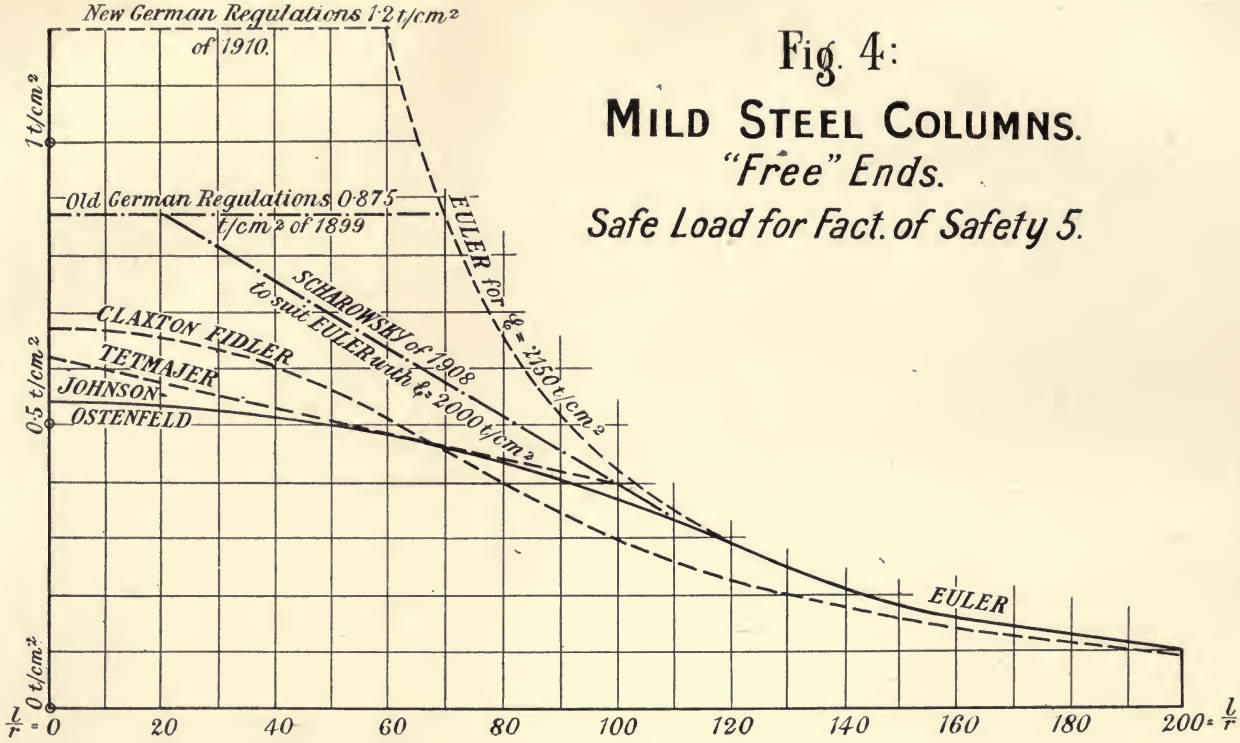

Fig. 4:

MILD STEEL COLUMNS.

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Safe Load for Fact. of Safety 5.

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